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## BLOCK FAILURE IN CONNECTIONS

### Including Effects of Eccentric Loads

Jeppe Jönsson

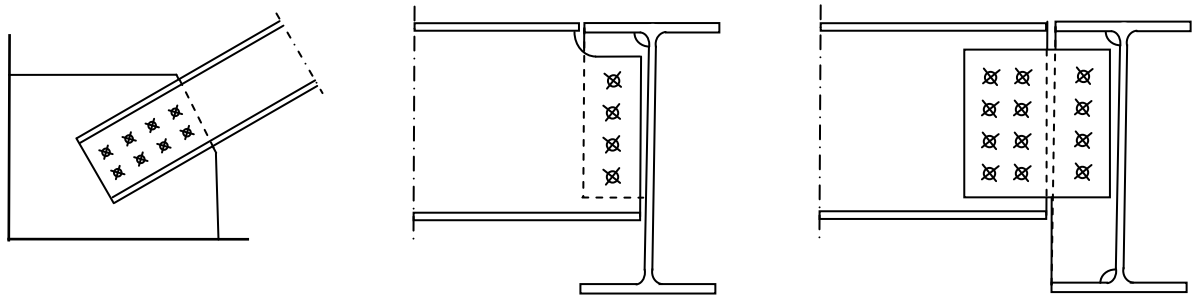
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## INTRODUCTION

Block tear-out in gusset and fin plate connections used for diagonal bracing, coped and non-coped beams is described in several codes as a block shear failure mechanism. However in many design situations the load to be transferred acts eccentrically or may involve the transfer of a substantial moment in combination with a shear force and/or a normal force. It is believed that the codes must include the moment effects from for example eccentricity of the shear load by appropriately and directly including all the section forces acting at the center of the individual bolt group or weld group of the connection.

In the present work a summary of some previous studies on block failure is given. The summary illustrates that no readily available tests with substantial eccentricity have been performed.



*Fig. 1.* Examples of gusset and fin plate connections.

Some theoretical work leading towards capacity methods including the combined influence of normal force, shear force and moment on the block failure capacity of gusset or fin plate connections will be exemplified. In fact plasticity based methods including the influence of the moment have previously been briefly described in a few steel design texts and practicing structural engineers normally do perform some additional design checks in order to verify the moment transfer capacity; especially when using Danish or German traditional beam gusset plate connections shown to the right in *Fig. 1*. The focus of the present work is to develop a few simple standardized capacity based methods involving the individual normal force, shear force and moment block failure capacities and a set of relevant capacity interaction formulas with a format related to those already in use for cross section analysis in the Eurocodes. In the presented work we use a lower bound method based on the assumption of rigid plastic behavior of the plate failing in a block failure mode. The block is usually a C-block cut-out or an L-block cut-out, however in this paper only the theoretical and experimental part on the C-block cut-out are treated. The failing block is surrounded by yield bands obeying the von Mises yield law. Due to varying strain hardening along the yield bands the final block failure mode may be calculated based on a maximum stress, which is a function of the material yield stress and the material ultimate stress, e.g. for example the mean value.

Experimental evidence of the rotational block failure of a combined shear and moment loaded gusset plate connection will be briefly presented. A completely new and statically determined and well defined test setup will be illustrated and some preliminary experimental results will be shown.

In the experimental study the fin plate or equivalent web plate and overall connection has been designed to fail in a block failure mode with interacting shear and moment load.

The focus of the present paper is entirely on block failure and we assume all other capacities (including bolt hole distances) proven to be adequate. It is the aim to illustrate the key features of the present research.

## 1 STUDIES ON BLOCK FAILURE

Investigation of stresses in gusset plate connections was reported in the research bulletin by Whitmore in 1952 [1]. Only concentric tension forces were applied even though the authors expected that bending stresses would have an influence on the failure mode. The paper suggests the so-called Whitmore method to design against the possibility of tearing out a block of the gusset plate, assuming a fictive tension failure line. The length (or width) of the fictive tension failure line was based on a 30 degree (1:2) spreading of the forces from the first to the last bolt line, see left illustration in Fig. 2.

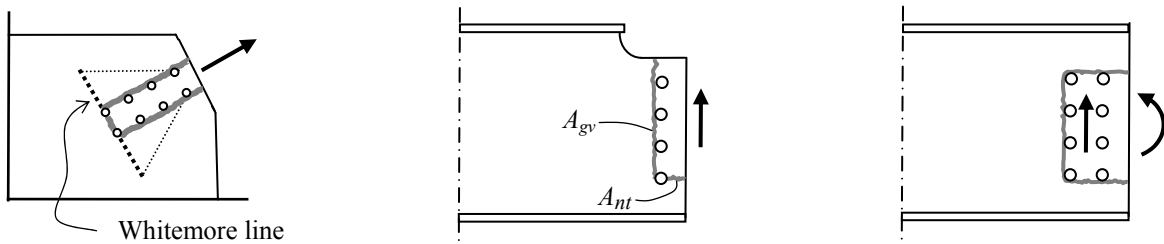


Fig. 2. Two conventional and an unconventional yield band block mechanism.

In the 1980s one of the papers which stand out is the one by Hardash & Bjorhovde in 1985 [2]. The paper suggested that the connection length should be a parameter for the block shear strength. This paper has been referred to by many and the equation for the block shear strength has been compared to experimental data also by others. The paper reports tests on 28 gusset plate specimens all with concentric load. Furthermore the paper includes results from 14 other tests, whereby in all 42 tests are used to verify the suggested equation for the ultimate capacity,  $V_R$ , which may be written as:

$$V_R = f_u A_{nt} + 0.575 f_{eff} A_{gv} \quad (1)$$

where  $A_{nt}$  and  $A_{gv}$  respectively are the net area in tension and the gross area in shear,  
 $f_{eff} = (1 - C_l) f_y + C_l f_u$ , where  $C_l = 0.95 - 0.047 \cdot l$  and  $l$  is the connection length in inches,  
 $f_u$  is the ultimate material strength,  
 $f_y$  is the yield strength of the material.

This is a formula for tension block capacity, which in contrast to Eurocode also includes the influence of the connection length, but otherwise it has the same format – for concentric loads.

In 2003 Franchuk et al. [3] investigated different parameters and their influence on the L-block tear-out shear capacity for coped beams. This paper supports previous papers that suggest that the tension contribution of the block shear capacity should be reduced by 50% for eccentric load due to non-uniform stress distribution. The experimental tests showed that the shear failure happened close to the gross shear area and it seems to document that the position of the block shear failure is to be based on the net area for the tension failure line  $A_{nt}$  and the gross area for the shear failure area  $A_{gv}$  as shown for the coped beam in Fig. 2. It is concluded that end rotation does not have a significant influence on the capacity. However the test setup is not representative for all types of fin or gusset plate beam connections.

Three papers [4], [5] and [6], published in the years 2001-2006 by a coinciding group of authors Kulak, Grondin, Huns and Driver, have used available experimental data in order to compare

experimental test results to different standards, equations suggested by other papers and their own suggested equations. The papers are different in their focus points and which equations they compare the test results to. In the first paper by Kulak & Grondin [4] the test results are collected and compared to the American, Canadian, European and Japanese Standard plus equations suggested in the paper. In the comparison the experimental data are categorized in connection types as gusset plates, angles, coped beams with one bolt line and coped beams with two bolt lines. In the second paper by Huns et al. [5] it is stated that it is important that the design equations reflect the failure mode and not only predict the capacity. The paper further states that the connection length does not have any influence on the capacity (for coped beams) even though it has been suggested and earlier shown for gusset plates.

*Table 1.* Constants for equation (2) from Driver et al [6]

Connection type	$R_t$	$R_v$
Gusset plates	1	1
Angles and tees	0.9	0.9
Coped beams: one bolt line	0.9	1
Coped beams: two bolt lines	0.3	1

In the third paper by Driver et al. [6] the objective was to compare the American, Canadian and European Standards plus an equation suggested by Topkaya [7] who as Hardash & Bjorhovde suggested that the connection length is a parameter and an equation by Cunningham et al. [8] with eccentricity as a parameter. This paper by Driver et al. also suggests an equation in which the position of the shear failure line is along the outer edge of the bolt holes, i.e. a gross length, as suggested by Franchuk et al. [3] and further that the shear strength is developing beyond the yield strength but not fully up to the ultimate strength. The paper comments on standards and published papers, which have not been able to agree upon a similar consistent approach towards the determination of the block shear strength. This paper also differentiates the experimental data into 3 separate categories: gusset plates, coped beams and angles. The suggested equation is the same for all types, however the constants depend on the connection type. The suggested equation for the resistance is:

$$V_R = R_t f_u A_{nt} + R_v \left( \frac{f_y + f_u}{2\sqrt{3}} \right) A_{gv} \quad (2)$$

where  $R_t$  and  $R_v$  are constants determined by the connection category as given in *Table 1*.

In another paper by Huns et al. [9] it is found that the shear capacity after tension rupture happens almost for the ultimate material strength. This indicates that the average between ultimate and yield strength suggested by Driver et al. [6] is conservative. This paper also concludes that the effect of the length of the connection is inconclusive and should therefore not be taken into account.

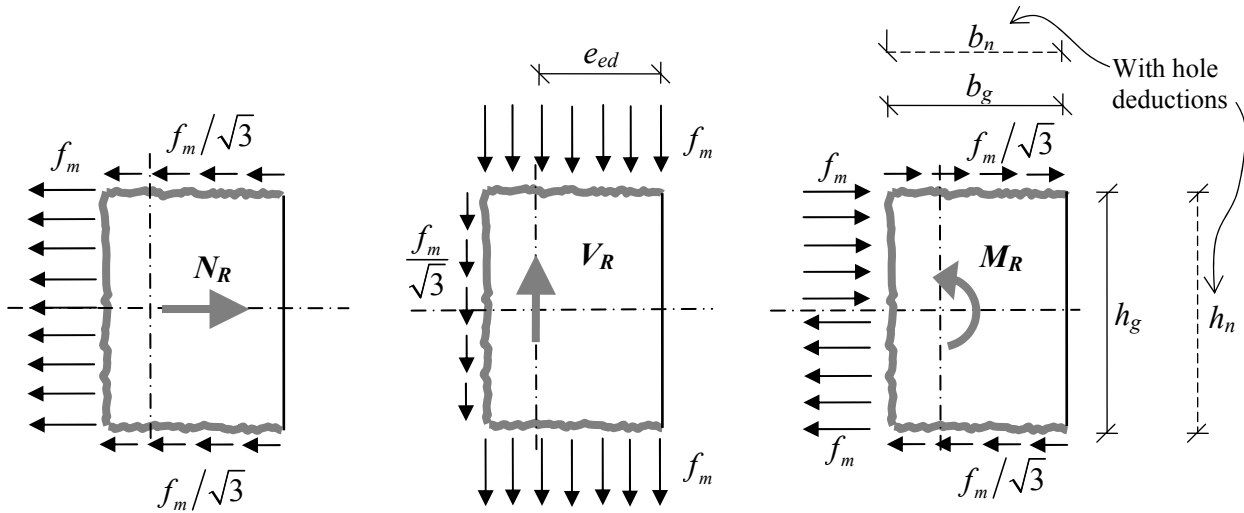
As mentioned, Topkaya suggested formulars for block shear capacity of angle connections in [7]. The paper contains results from FE analysis of experimental block shear tests in order to develop expressions that describe the actual failure. The paper suggests that the strength of the shear plane is dependent on the length of the connection and that the strength is dependent on a ratio between the yield and ultimate strength. All three formulas proposed are for concentric loads and the paper suggest a factor that reduces the strength by 10% if the load is eccentric.

To sum up the findings in the literature, the crucial conclusion is that none of the papers have been dealing with complex loading including substantial eccentricities and general block failure. It is stunning that the stress distributions in equilibrium with sections forces have not been investigated; perhaps they have been abandoned due to the seemingly complex stress distributions. The reason could be that the current formulas are based on empirical research instead of relating it to a theoretic approach based on plasticity theory and yield bands. It seems very inappropriate just to introduce a

reduction factor of 0.5 on the tension area in the L-shaped block failure modes of the codes in order to accommodate eccentric loading with its magnitude remaining unquantified.

## 2 CAPACITY METHODS FOR RIGID-PLASTIC BLOCK FAILURE ANALYSIS

Assuming that the material behaves as a rigid plastic material and that all material outside the yield bands is rigid opens the possibility of determining the work performed during deformation of the rigid blocks relative to each other. This leads to an upper bound for the capacity of the given load situation, this involves numerical integration along the yield lines and superposition of results are not possible. However lower bound plasticity methods may lead to the formulation of relatively simple approximate methods relating individual block failure capacities for normal force, shear force, and moment as well as the interaction of these by simple von Mises based interaction formulas as exemplified in the following. The lower bound methods are based on the assumption that the equilibrium stress fields of the rigid parts are below or at yield everywhere. Thus to find a good lower bound approximation we have to find effective allowable stress fields along the yield bands. Three basic stress fields acting along the yield band (shown without holes) on the cut-out block are shown in *Fig. 3*. The three relatively simple situations for a C-shaped cut-out correspond to the normal force  $N_R$ , the shear force  $V_R$  and the moment  $M_R$  block mechanism capacities. Based on experimental observations it is assumed that the holes are just inside the block and the gross dimensions are given by  $h_g$  and  $b_g$ . Deductions for holes are given by the net lengths respectively corresponding to  $h_n$  and  $b_n$ . Furthermore it is assumed that the normal stresses are acting on the net yield band lengths and shear stresses on the gross yield band lengths. (During mechanism forming and yielding the bolt holes are elongated and leave an open trace). The shear force is acting at the shear distribution center, given by the edge eccentricity  $e_{ed}$  as shown in the central situation of *Fig. 3*. The straining of the yield bands varies along each line and strain hardening will commence before the yield mechanism has fully formed, therefore a formal yield stress  $f_m$  is used. A value of the formal yield stress of  $f_m = (f_y + f_u)/2$  corresponding to the mean value of the yield stress and the ultimate stress is used. (The shown distributions are relevant for  $b_n \leq 1.15h_g$ ).



*Fig. 3.* Yield band stress actions on the block for three basic C cut-out block mechanisms.

The individual capacities can now be determined by the following relatively simple formulae:

$$N_R = t f_m \left( 2 \frac{b_g}{\sqrt{3}} + h_n \right), \quad V_R = t f_m \left( 2 b_n + \frac{h_g}{\sqrt{3}} \right), \quad M_R = t h_g f_m \left( \frac{b_g}{\sqrt{3}} + \frac{h_n}{4} \right) \quad (3)$$

It is possible to combine factored basic stress fields in order to derive stress fields for interactive situations. If we factor each of the three stress fields with respectively  $N/N_R$ ,  $V/V_R$  and  $M/M_R$ , we can combine the stress fields, such that we obey the Von Mises yield condition along each yield

band. It turns out that with the three given basic mechanisms we just need to fulfil the following simple interaction equation:

$$\left(\frac{N}{N_R} + \frac{M}{M_R}\right)^2 + \left(\frac{V}{V_R}\right)^2 \leq 1 \quad (4)$$

where  $N$ ,  $V$  and  $M$  are the section forces at the shear distribution centre given by:

$$e_{ed} = \frac{b_g b_n + b_g h_n / \sqrt{3}}{2b_n + h_g / \sqrt{3}} \quad (5)$$

There are several other possible ways of distributing the stresses along the yield lines in order to get a higher lower bound carrying capacity. Some of these distributions have been investigated in the master thesis by Nørgaard [10]. The individual capacities of the basic block mechanisms given in Eq. (3) are all lower bounds (with the given assumptions) and the interaction Eq. (4) also corresponds to a lower bound capacity of a combination of the three basic mechanisms. In the following section an example of the accuracy achieved in comparison to one (of twelve) experimental result will be given and illustrated.

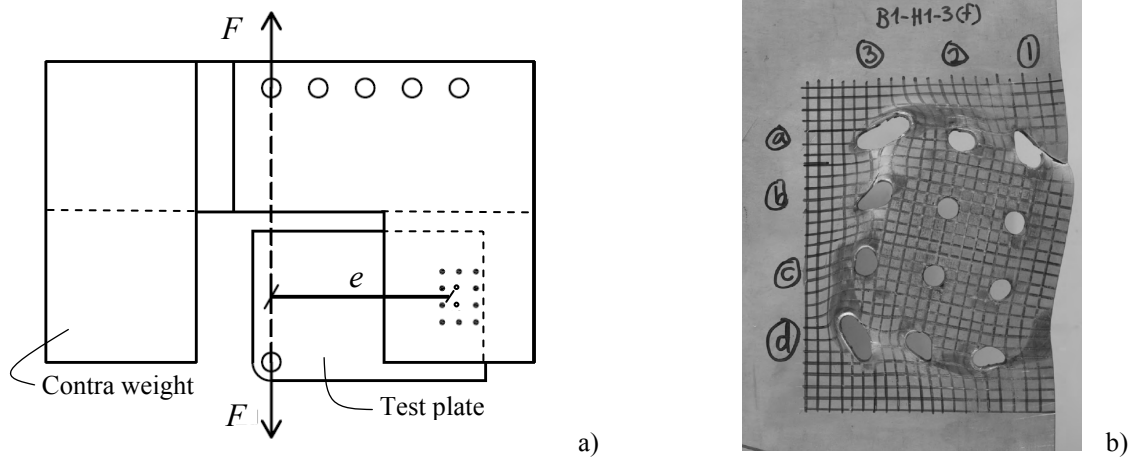


Fig. 4. a) Experimental test setup. b) Photo of block failure in the test plate after test.

Table 2. Results from one experiment and results of lower bound equations using the given parameters.

Test	$h_g$	$h_n$	$b_g$	$b_n$	$e_{ed}$	$f_y$	$f_u$	$f_m$	$V_R$	$M_R$	$e$	$F_R$	$F_{exp,y}$	$F_{exp,u}$	$F_R/F_{exp,y}$
	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[MPa]	[MPa]	[kN]	[kNm]	[mm]	[kN]	[kN]	[kN]	[-]
B1-H1-3(f)	138	84	122	82	81	272	375	324	782	40.8	393	103	129	196	0.80

### 3 EXPERIMENTAL INVESTIGATION OF GENERAL BLOCK FAILURE

In the B.Eng. project and thesis by Nissen [11] an experimental test setup has been designed and block failure experiments on bolted connections have been performed. The connections have been designed in order to assure that block failure was the decisive mode of failure. Twelve tests were performed on three different loading configurations with the same bolt group of 4x3 M12 bolts (10.9). In this paper only the representative test shown in Fig. 4. will be treated. In this test the experimental loading corresponds to zero normal force  $N=0$ , a shear force of  $V=F_R$  applied with a fairly large eccentricity  $e$  in relation to the shear distribution centre resulting in a connection moment of  $M=eF_R$ . The lower bound capacity found using Eq. (4) becomes

$$F_R = \left[ \left( \frac{e}{M_R} \right)^2 + \left( \frac{1}{V_R} \right)^2 \right]^{-0.5} \quad (6)$$

The experimental mechanism yield load  $F_{exp,y}$  is determined as the crossing point of the observed linear initial inclination (stiffness) and a tangent line with 10% of this inclination just touching the upper part of the curve (corresponding to 10% hardening), see Moore et al [12]. The ultimate capacity is just the maximum achieved load  $F_{exp,u}$ . The important parameters, results from lower bound equations and results from one experimental investigation are given in Table 2. The applied test load verses the piston movement and verses approximate angle of rotation are shown in Fig. 5.

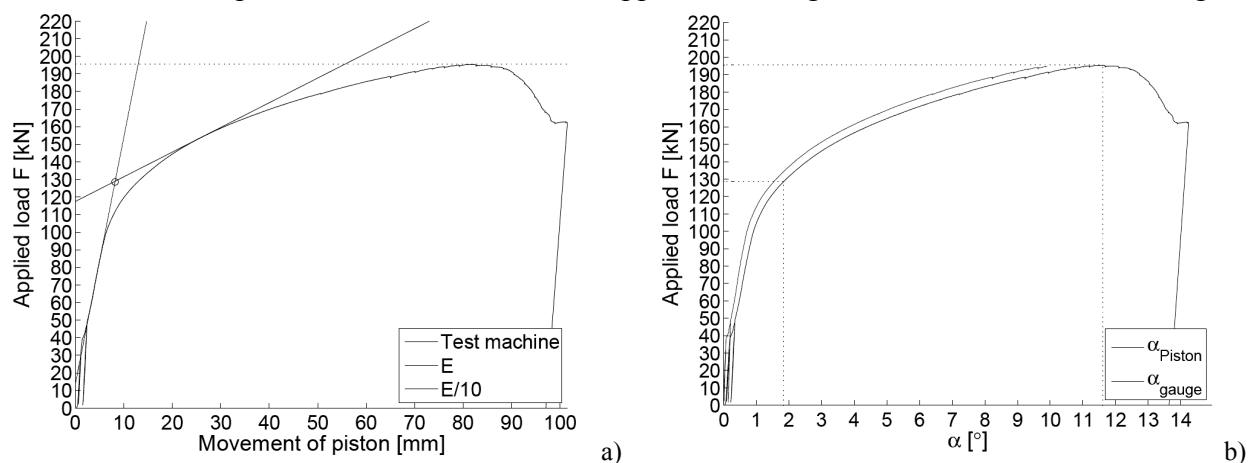


Fig. 5. a) Applied load verses piston movement; b) Applied load verses the angle of rotation.

## 4 CONCLUSION

The theoretical and soon experimental background for including effects of eccentric loads in block failure has been presented and ought to be included or allowed for in the codes.

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